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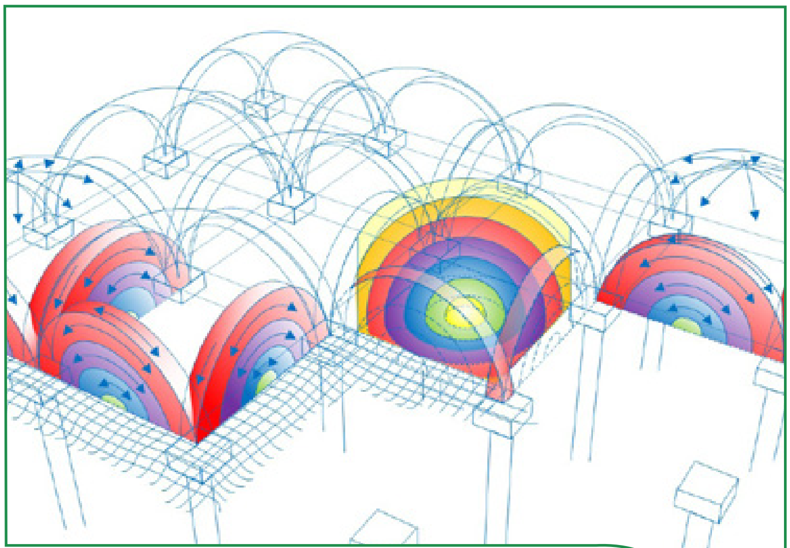

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The NGO welcomes you to the EuroGeo6

Dear conference guest,

The Dutch chapter of the IGS (NGO) welcomes you to the 6th European Geosynthetics Congres. As IGS Chapter, we would like to take this opportunity to introduce you to the NGO, by means of this Conference Special edition of GeoArt.

The NGO (Nederland Geotextielen Organisatie) was founded in December 1983 and therefore even precedes the IGS. From the time the IGS was established the NGO became the Dutch Chapter. NGO has always been an active chapter of IGS. Our goal is to "Promote the responsible use of geosynthetics". We put a lot of effort onto PR. We have our website www.ngo.nl which is directly linked to the IGS site and our own magazine GeoKunst. "Kunst" means art or artificial. So the pun (GeoKunst / GeoArt) works in both languages. GeoKunst is published 4 times per year as an insert in GeoTechniek. 4000 copies of this magazine are distributed to just about everyone in The Netherlands and Flemish Belgium, who is interested in geotechnical engineering. We publish 2 full length articles in each edition. NGO organizes an annual NGO meeting to which we invite key note speakers from the Ministry of Public Works, engineering companies, contractors and producers of geosynthetics to present papers and to share their thoughts and experiences on developments in geosynthetics and the use of geosynthetics in civil engineering projects. Another typical activity we organize annually is our Geosynthetics Workshop. Each year a theme is selected: dyke construction, piled embankments, slope stability, etc. The workshop consists of a number of presentations and always ends with a challenge. Teams are formed and each team is asked to build a scale model of their solution to the challenge, with the materials provided (figure 1). The models are then "tested" and a much coveted prize is awarded to the winning team.

Figure 1 **-** *The challenge 2016: Test to piping failure of a dyke construction.*

The Netherlands is a delta area and basin to the Rhine and the Muse rivers, which branch out in the Netherlands and finally flow into the North Sea near Rotterdam. Much of middle and west of the country is below sea level and has highly compressive soft soils (peat and clay) which makes life challenging when building infrastructure. The foundations of most buildings, bridges and viaducts in these areas are piled. Most roads and railroads are not, however the piled embankment is steadily increasing in popularity, as this has proved its self to be a highly effective construction method for building raised infrastructure on soft soils, especially as embankment to piled viaducts or bridges. Recently the Dutch Design Guideline CUR226 for basal reinforced piled embankments was revised and published in it's second edition in Dutch and English. One of the main changes was the adoption of the Concentric Arches method of Van Eekelen (figure 2). The new design method is presented in the article "The 2016-update of the Dutch Design Guideline for Basal Reinforced Piled Embankments" by Suzanne Eekelen.

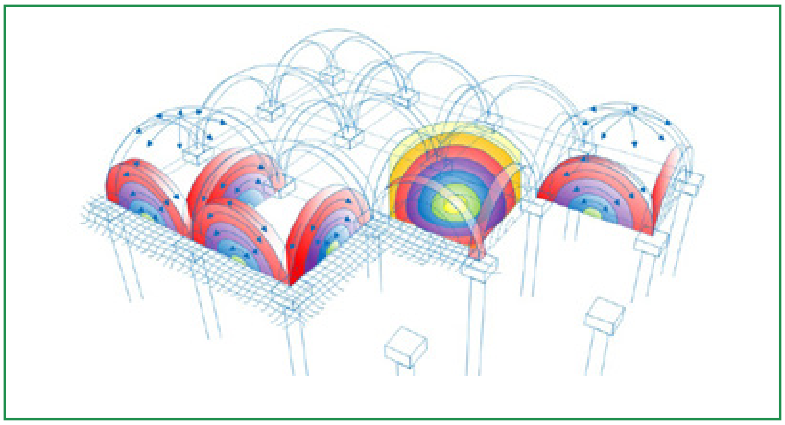


Figure 2 - *Concentric arches in a piled embankment.*

An overview of risks and lessons learned with regard to geomembrane systems in The Netherlands are given in the article "Geomembrane systems in the Netherlands and abroad – Risks and Lessons-learned" by Rijk Gerritsen, Charlie Angenent and John Scheirs. Subjects addressed in this article include the methods of risk analysis regarding geomembranes, quantification of risks, risk-assessments of installation methods, welding procedures and methods, design aspects, materials, monitoring, inspection methods (figure 3). The aim of this article is to emphasize an integral approach to the application of geomembranes and the importance of acknowledging and understanding quality risks by all stakeholders. The success of realizing a watertight and durable sealing depends on a combination of good understanding of the design aspects and the materials and on quality assurance during the construction process.

Geosynthetic requirements for basal reinforcement are dependent on the application and should be written with the data characteristics in mind. The product choice should not be driven by the production technology, but only by the conformity of the product to the characteristics required for the design phase as explained in Alain Nancey and Dick Janse's article "High strength wovens, effective and economic geosynthetics for basal reinforcement". In this article, the geosynthetic requirements for the reinforcement function are discussed as well as their impact on the behaviour of structures where basal reinforcement is involved.

We hope you enjoy the conference, that you make good use of the opportunity to expand your network and that you return home with new ideas and enthusiasm for the many innovative and boundary breaking solutions to every day challenges.

Warm regards,

Erik Kwast: Editor in chief GeoKunst / Chairman of the NGO.

The 2016-update of the Dutch Design Guideline for Basal Reinforced Piled Embankments

dr. Suzanne J.M. van Eekelen
Deltares, Chair Dutch Committee Design Guideline Basal Reinforced Piled Embankments CUR226, the Netherlands

1 Introduction

The first piled embankment reinforced with geosynthetic reinforcement (GR) was constructed in 1972 in the in de Göta älv valley in Sweden (Holtz en Massarsch, 1976). The first British piled embankments were constructed in the early 1980s, one of them as the foundation of an abutment of the Second Severn Crossing (Russell and Pierpoint, 1997). The Monnickendam Bus Lane is the first Dutch piled embankment, dating back to 2000. Since those days many basal reinforced piled embankments have been constructed worldwide.

A basal reinforced piled embankment (Figure 1) consists of a reinforced embankment on a pile foundation. The reinforcement consists of one or more horizontal layers of geosynthetic reinforcement (GR) installed at the base of the embankment. The embankment is filled with for example crushed demolition recycled aggregate (hard core) or sand.

A basal reinforced piled embankment can be used for the construction of roads or railways when a traditional construction method would require too much construction time, affect vulnerable objects nearby or give too much residual settlement, making frequent maintenance necessary. Some piled embankments are long, like the Dutch 14 km long regional road N210 between Krimpen and Bergambacht, or the Dutch 3.5 km long bypass road near Reeuwijk (Van Eekelen and Venmans, 2016). Other piled embankments are short and for example constructed for transition zones between traditional embankments on soft soils and a fixed structure or as an abutment for a viaduct.

The first edition of the Dutch design guideline for basal reinforced piled embankments CUR226 was published in 2010 (Figure 4, describted in Van Eekelen et al., 2010b) and adopted major parts from the German EBGEO (2010). Since those days, the knowledge about basal reinforced piled embankments has developed substantially; a more reliable design method for the GR became available (Van Eekelen et al., 2015 and Van Eekelen, 2015), an adaptation to the Eurocode was needed and questions about pile cap design arose, making an update of CUR226 necessary. This paper describes the highlights of the 2016-update of CUR226.

2 Geosynthetic Reinforcement (GR) Design

The GR strain needs to be calculated to design the GR. Multiplying this GR strain by the GR stiffness gives the tensile force, which needs to be smaller than the long-term GR tensile strength. Most calculation models calculate the GR strain in two steps (Figure 2a and b). Step 1 divides the vertical load into two load parts. One part (load part *A*) is transferred to the piles directly. This part is relatively large because a load tends to be transferred to the stiffer parts of a construction. This mechanism is known as 'arching'. The second, residual load part (*B*+*C*) rests on the GR (*B*) and the underlying subsoil (C), see Figure 2c.

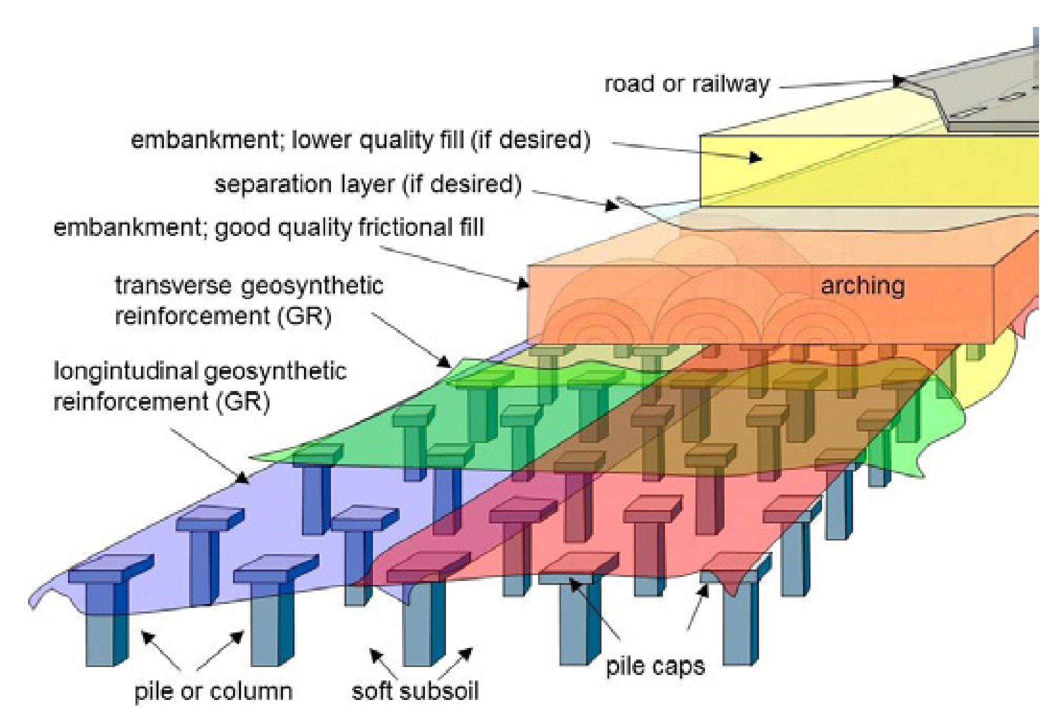
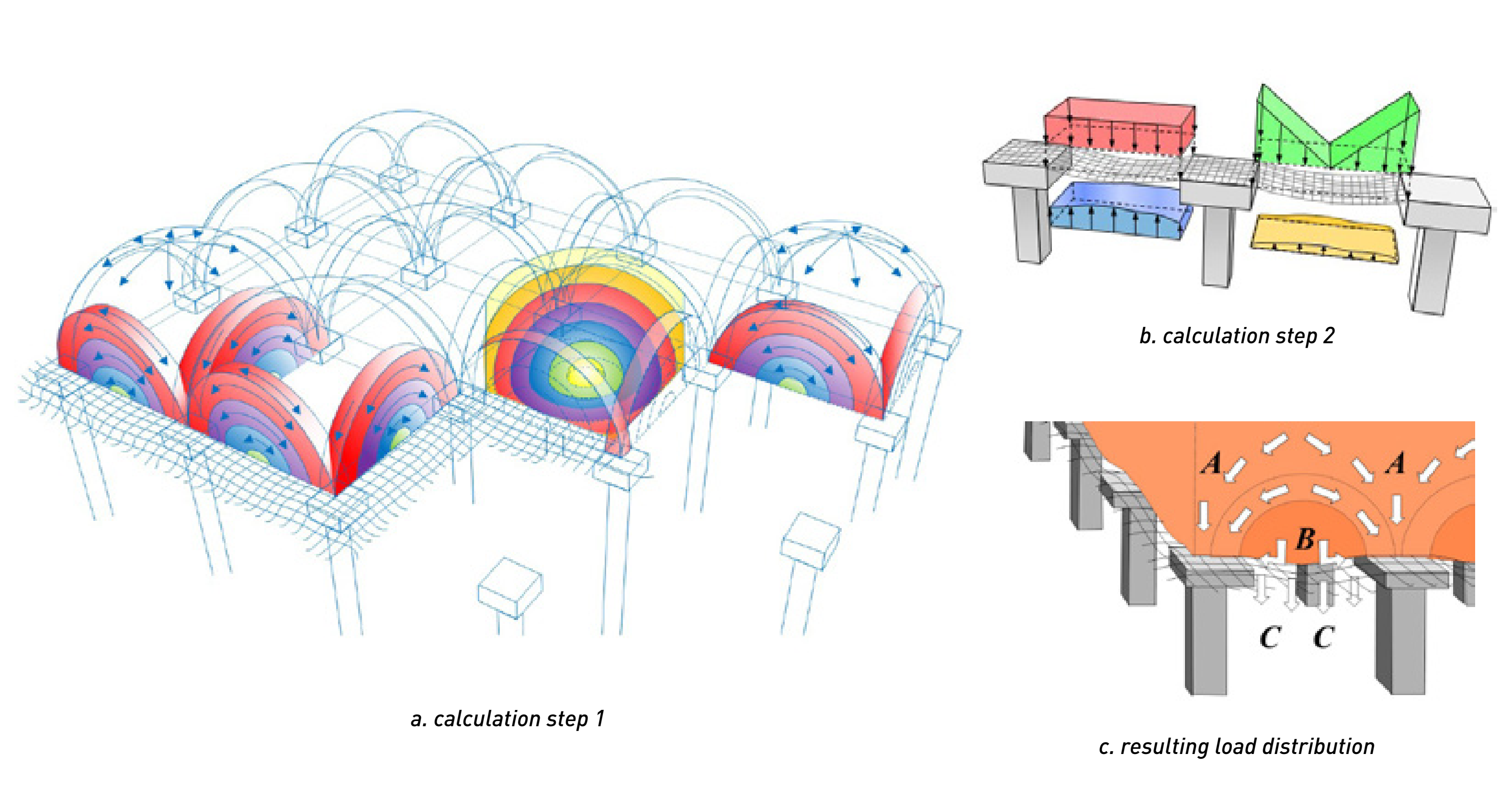


Figure 1 - *A basal reinforced piled embankment.*

Abstract

The Dutch Design Guideline CUR226 for basal reinforced piled embankments was revised before publishing its second edition in Dutch and English in 2016. This paper reports about the main changes in comparison to its first edition, of 2010. One of the main changes was adopting the Concentric Arches method of Van Eekelen et al. (2012b, 2013, 2015) and van Eekelen (2015) for the design of the geosynthetic reinforcement (GR). An accompanying set of partial safety factors was derived with an extensive probabilistic study. The Eurocode guidelines for traffic load were adopted and converted into a uniformly distributed design load for piled embankments using the Boussinesq spreading method. This resulted in a practical set of tables. Finally, a design guideline for the pile caps on top of the piles was added. Extensive calculation examples support the use of the new guideline.



a. calculation step 1

b. calculation step 2

c. resulting load distribution

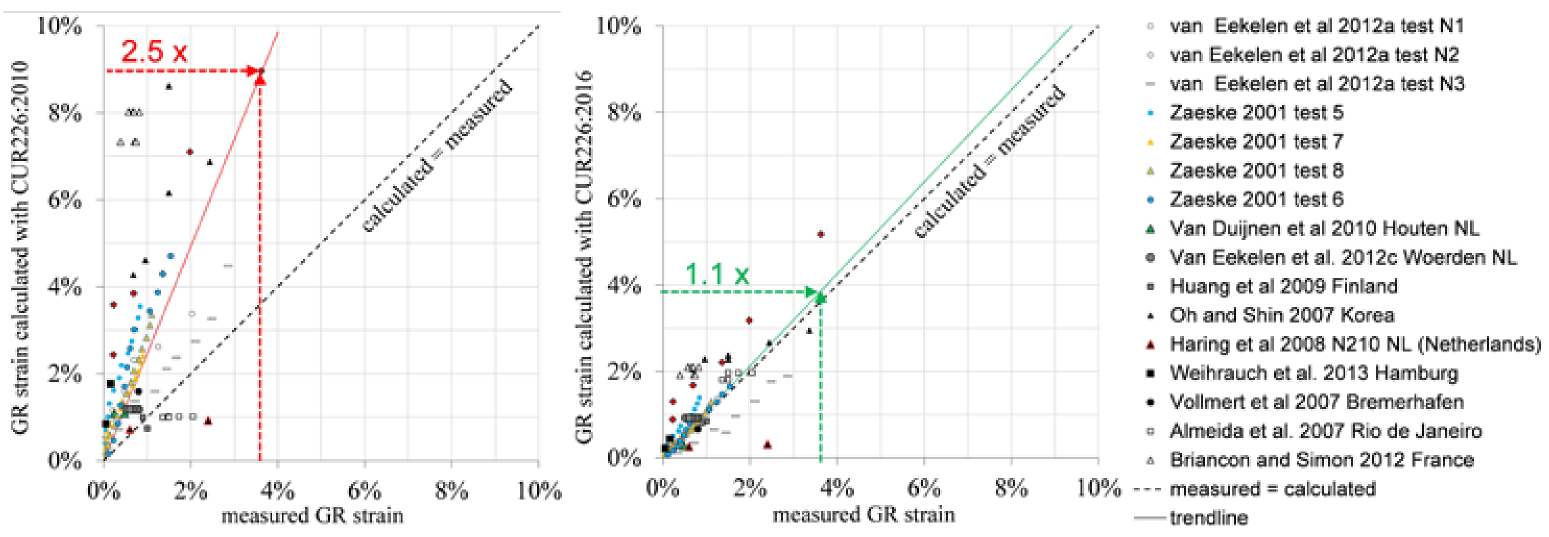
Figure 2 - *The new Concentric Arches model (Van Eekelen, 2015 and Van Eekelen et al., 2012b, 2013, 2015) consists of two steps: (a) step 1 calculates the load distribution in A and B+C and (b) step 2 calculates the GR strain that occures in the GR strip between adjacent pile caps (c) resulting load distribution (A, B, C).*

Calculation step 2 determines the GR strain. Only the GR strips between each pair of adjacent piles are considered: they are loaded by *B+C* and may or may not be supported by the subsoil. The GR strain can be calculated if the distribution of load part *B+C* on the GR strip, the amount of subsoil support and the GR stiffness are known.

CUR226 (2010) used the calculation model of Zaeske (2001) for GR design. The German EBGEO had already adopted that model before. The Dutch made the same choice for Zaeske's model because it matched some field measure-ments reasonably well. However, many more measurements became available since 2010. Van Eekelen et al. (2015) showed that Zaeske's model gives on average 2.5 times the strain measured in seven field cases and four laboratory series of experiments (Figure 3a).

CUR226 (2016) uses the Concentric Arches model of Van Eekelen (2015) and Van Eekelen et al. (2013, 2015). This model was developed on the basis of a series of laboratory tests (Van Eekelen et al., 2012a and 2012b). Calculation step 1 consists of a set of 3D and 2D concentric arches as shown in Figure 2a. The load is transported along the concentric arches. Smaller arches exert less load on their subsurface, large arches exert more load on their subsurface. The result is that a relatively large load is exerted on the pile caps (*A*) and the GR strips between adjacent piles, which matches measurements quite well. Figure 2b shows the load distribution on the GR strips between adjacent piles for step 2 as adopted in CUR226 (2016); when there is no subsoil support, or almost no subsoil support, the inverse triangular load distribution is used. When there is significant subsoil support, a uniform load distribution is used.

Figure 3b shows that the GR strain calculated with the new model is on average 1.1 times the measured GR strain with a lower coefficient of determination, R2, than shown in Figure 3a. The calculated GR strain is therefore almost a perfect match with the measured GR strain. CUR226 (2016) has therefore adopted the Concentric Arches model.



a. calculations with CUR226:2010 *b. calculations with CUR226:2016*

Figure 3 **-** *Comparison calculations and measurements in seven field projects and four series of experiments. Van Eekelen et al., 2015 gives the sources of the references given in this picture, which are not given in the references of this paper due to space limitations. Calculations without safety factors.*

3 Model Factor, Load and Material Partial Factors

One can debate whether a design guideline should adopt a model that nearly always gives a design on the safe side, as with the Zaeske model (Figure 3a), or whether a design guideline should adopt a model that describes reality as well as possible (Figure 3b) and consider safety separately. The Dutch CUR226 committee decided to adopt the new Concentric Arches model and to combine this with the inclusion of a model factor to cope with the uncertainty in the model. The value of the model factor was determined using the data points given in (Figure 3b).

Van Duijnen et al. (2015) reported the safety analysis used to determine the model factor and the associated load- and material factors. Following the suggestions made in EC1990 (2011, Eurocode 0), they conducted a statistical assessment of the differences between the measured and calculated GR strains and then carried out Monte Carlo (MC) simulations for the SLS situation, for several reference cases in order to obtain the model factor. Multiplying this model factor by the GR strain calculated with characteristic values gives a value that is higher than the real GR strain in 95% of the cases. In other words, if the model factor is used, reality is worse than the calculation in 5% of the cases.

Table 1 - Model factor and partial safety factors used for the design of the GR design in CUR226 (2016).

		SLS	Reliability class ULS		
			RC1	RC2	RC3
	Factor	$\beta \geq 2.8$	$\beta \geq 3.5$	$\beta \geq 4.0$	$\beta \geq 4.6$
Model factor	γ_M	1.40	1.40	1.40	1.40
Traffic load p	$\gamma_{f;p}$	1.00	1.05	1.10	1.20
Tangent of internal friction, tan φ'	$\gamma_{m;\varphi}$	1.00	1.05	1.10	1.15
Unit weight fill, γ	$\gamma_{m;\gamma}$	1.00	0.95	0.90	0.85
Subgrade reaction of subsoil, k_s	$\gamma_{m;k}$	1.00	1.30	1.30	1.30
Axial GR stiffness, J	$\gamma_{m;EA}$	1.00	1.00	1.00	1.00
GR Strength, T_r	$\gamma_{m;T}$	1.00	1.30	1.35	1.45

The calculated strain should be multiplied with the model factor γ_M, γ_f is a load factor, $F_d = \gamma_f \bullet F_k$, γ_m is a material factor, $X_d = X_k / \gamma_m$, a unit weight increase is not beneficial, hence the value of $\gamma_{m;\gamma}$ is less than 1.0.

Subsequently, Van Duijnen et al. (2015) determined three sets of partial material and load factors associated with the model factor for a level 1 design approach (the method with partial factors). They showed that using these factor sets satisfy the reliability indices β required for the three reliability classes of EC1990 (2011, Eurocode 0). The resulting model and partial factors were adopted in CUR226 (2016) and are shown in Table 1.

Extensive calculation examples of GR design were included in the 2016-update of CUR226.

4 Limitations

CUR226(2016) was written for piled embankments with a basal geosynthetic reinforcement. The validation of the GR design rules was conducted with measurements in piled embankments with:

Table 2 - Maximum average uniformly distributed traffic load $p_{traffic}$, based on NEN-EN 1991-2 for number of passages per year: N = 2.000.000, 2 driving lanes, with driving lane 1 heavy traffic: 4 wheels F_{wheel} = 120 kN and $q_{uniform}$ = 7.2 kPa and the second driving lane: 4 wheels with F_{wheel} = 100 kN and $q_{uniform}$ = 2.5 kPa.

	Pile spacing			
Height embankment H [m]	1.0 · 1.0 m²	1.5 · 1.5 m²	2.0 · 2.0 m²	2.5 · 2.5 m²
1.0	74.99	70.66	62.11	52.78
2.0	44.04	41.94	39.43	36.77
3.0	28.80	28.01	27.04	25.94

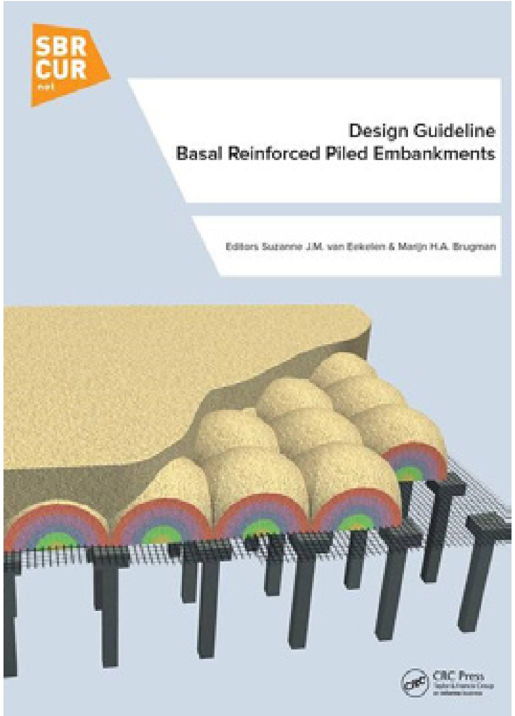


Figure 4 - *The 2016 update of CUR226.*

- a centre-to-centre (ctc)-pile spacing < 2.50 m;
- geogrids, in some cases combined with woven geotextiles (geogrid on top of geotextile);
- a groundwater level below or only just above the pile caps;
- $0.5 < H/(s_d - d_{eq}) < 4.0$; with H (m) the height of the embankment, s_d (m) the diagonal ctc-pile spacing and d_{eq} (m) is the diameter or equivalent diameter of the pile cap;
- vertical stresses on top of the GR above the pile caps up to 1450 kPa. In practice, however, some embankments of this type have already been realised with vertical stresses on the pile cap of 2000 kPa.

Furthermore, CUR226 (2016) gives the following limitations for its applicability:

- $H/(s_d-d_{eq}) \geq 0.66$ with H, s_d and d_{eq} explained above ;
- $p_{traffic} < p_{embankment\ weight}$ or apply κ-model of Heitz (2006, section 6), see section 5 for $p_{traffic}$;
- $b_{eq}/s_{x,y} \geq 0.15$ with b_{eq} the width of a square pile cap or the equivalent width of a circular one;
- one GR layer: $z \leq 0.15$ m, two GR layers: distance between two layers ≤ 0.20 m with z (m) is the distance between GR and pile cap;
- $2/3 \leq s_x/s_y \leq 3/2$;
- $\varphi'_{fill,cv} \geq 35^o$ for the lowest layer with height $h^* = 0{,}66(s_d-d_{eq})$. Above that, $\varphi'_{fill,cv} \geq 30^o$;
- $T_{r,d} \geq 30$ kPa, in both directions, and $0.1 \leq T_{r;x;d}/T_{r;y;d} \leq 10$ where $T_{r,d}$ (kN/m) is the short term GR tensile strength;
- $k_{s;paal}/k_{s;subsoil} > 10$, with k the subgrade reaction.

5 Traffic Load

The traffic loads given in load model BM 1 of EC1991-2 were included in the CUR226 (2016). These loads were converted into a uniformly distributed load, resulting in tables with values that were determined as follows:

- the axle loads were spread according Boussinesq over the total height of the embankment;
- the influence of all wheel loads were summed;
- determination of the average stress ptraffic on the maximum loaded pile grid ($s_x \cdot s_y$), with $s_{x,y}$ (m) the ctc-pile spacing.

Table 2 presents a summary of a larger CUR226 (2016) table, which gives more tables for smaller values for N and for the situation with only one driving lane. When using these tables, the extra spreading capacity of the asphalt top layer may be taken into account with a virtual extra height.

6 Heavy Traffic, Thin Embankment

Heavy truck passages influence the arching in the embankment, specifically in a shallow embankment. Van Eekelen et al. (2010a) showed that the arching is reduced as a result of a heavy passage. This results in a temporarily increase of the vertical load on the GR. They also showed that arching recovered during a rest period after a number of passages. Heitz (2006) conducted experiments with high dynamic loading on a test set-up with four small square piles underneath a sand embankment with and without a geosynthetic basal reinforcement. He found that (1) the arching reduces due to dynamic loading; (2) the arching recovers during a rest period and (3) cyclic loading has significantly less influence on the arching in (a) a relatively thick embankment or (b) an embankment with GR in comparison to one without GR.

On the basis of his unreinforced experiments, Heitz (2006) determined an empirical model to reduce arching, the so-called κ (kappa)-model. This κ-model is at the safe side as Heitz based his model on his unreinforced experiments, which are the tests with the heaviest influence on the arching.

The Heitz-κ model had already been included in CUR226 (2010). However, the graphs determining the κ-values were modified and brought in line with the original ideas of Heitz.

7 Pile Design

CUR226 (2010) assumed that an embankment is not able to re-distribute the load in the case of pile failure (a non-stiff construction). In CUR226 (2016), this rule has been extended. If the embankment has enough height: $H/(2 \cdot s - d) \geq 0.66$, the construction may be considered as stiff and the embankment is assumed to be able to re-distribute the load in the case of pile failure. In this equation is H (m) the height of the embankment, s_d (m) the diagonal ctc-pile spacing and d_{eq} (m) the diameter of circular pile caps or the equivalent diameter of square pile caps.

When the reinforced embankment is non-stiff, which will often be the case for shallow embankments where the pile spacing is maximised, the design should be done using the factors appropriate for a non-stiff structure, unless it has been demonstrated that the structure will continue to perform if one pile fails.

The other calculation rules for the geotechnical bearing capacity of the piles follow the normal local design guidelines (NEN 9997-1 in the Netherlands). Pile moments and cross forces must be calculated with a numerical program using for example finite element analysis, which is further explained in Section 9.

8 Pile Cap Design

Pile caps are circular or square and preferably have rounded edges to prevent GR damage. If pile caps have sharp edges, protection

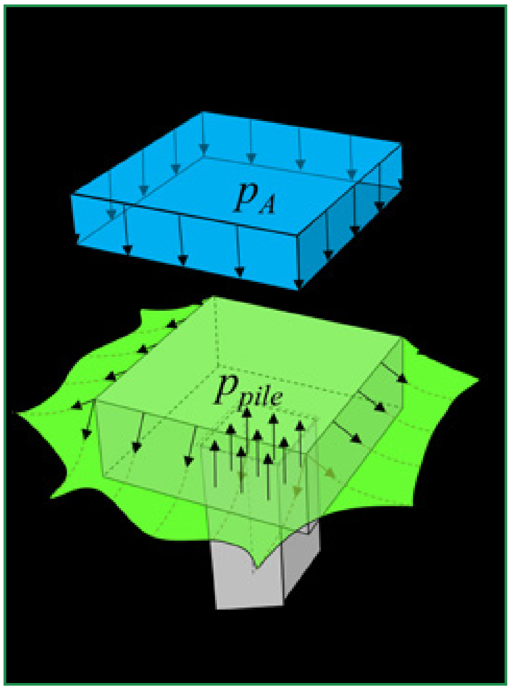


Figure 5 - *Loads on a pile cap: arching p_A (kPa), the tensile force T (kN/m) and the axial pile force p_{pile}, uniformly distributed (kPa).*

measures are recommended to protect the GR against damage. A square pile cap size frequently applied in the Netherlands is 0.75m · 0.75m · 0.30m; which is small and thick in comparison to 'normal' concrete structures, this height is usually enough to span several meters. CUR226 (2016) specifies the loads on the pile caps as follows (Figure 5):

- vertical load (*A*) due to arching (Figure 2c), uniformly distributed on the pile cap (kPa);
- tensile force *T* from the geosynthetic reinforcement (kN/m);
- axial pile force from underneath, which is assumed to be uniformly distributed (kPa).

The pile cap should be checked on punching and bending. Usually, the thickness of the pile cap is enough to prevent punching. The strength of the steel reinforcement in the pile cap needs to be enough in the ULS and the sustainability, or the crack width, should be enough in the SLS. The English version of CUR226 (2016) gives a summary of the more lengthy Dutch pile caps chapter.

9 Numerical Calculations

The GR design should be carried out analytically with the Concentric Arches model. CUR226 (2016) does not allow numerical GR design. However, numerical calculations are usually necessary to determine deformations, pile moments and cross forces. With numerical calculations, the influence is determined:

- on adjacent objects;
- of adjacent existing and future objects;
- of lateral loads such as traffic, spreading forces in the embankment.

In daily practice, 2D calculations are generally used, both in longitudinal and transverse direction. The 3D appearance of piled embankments makes it necessary to consider carefully the pile stiffness, the soil behaviour between piles, pile settlement behaviour and vector summing of pile moments and cross forces.

All relevant construction stages and secondary effects need to be included. A 'gap' needs to be applied between subsoil and GR in the cases that the subsoil support will disappear during service life.

For each cross section, two numerical calculations are needed. The first is conducted with calculation values, although it is an option to use calculation values in the normative phase only, and characteristic values in the other phases. The second calculation must be conducted with characteristic values. The results of the second calculation should be multiplied with 1.2 and should then be compared with the results of the first calculation. The highest values are the normative pile moments and cross forces.

10 Conclusions

This paper presents the 2016-update of the Dutch design guideline for basal reinforced piled embankments. This guideline has been developed in full compliance with the Eurocodes, including Eurocode 7 (NEN-EN 1997-1) with its national appendices. The guideline has been published both in Dutch and English.

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Basal reinforcement with high strength geosynthetics

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1 Introduction

Since more than two decades, the ability of high strength geosynthetics to improve significantly the stability and the cost efficiency of the whole structure has been demonstrated. Embankments on soft soils, on piles or bridging voids are applications where geosynthetics are used as basal reinforcement.

A proper design of a geosynthetic solution should establish the link between the measurable characteristics of the product and the performance expected for the structure. Particularly in Europe, the application standard EN13251 gives the characteristics of geotextiles and geotextile related products required for use in earthworks, foundations and retaining structures. Design standards and guidelines allow calculating the design values for each relevant characteristic. Strength, stiffness, soil interaction or hydraulic properties are characteristics that need to be verified by measurement on the product.

1.1 Requirements from application standard

Table 1 gives the main characteristics of geosynthetics required for the reinforcement function. Reinforcement is indeed the main function of the geosynthetic in basal reinforcement, but sepa-ration or filtration can be required as secondary functions, particularly when the geosynthetic is between two layers of different particle sized materials.

1.2 Requirements from design standards: reduction factors

Tensile strength, elongation at maximum load and stiffness at 2%, 5% and 10% are characteristics used directly in the calculations to ensure the stability and serviceability of the structure. All characteristics related to the durability needed to assess the long term behaviour of the product are expressed as a reduction factor. They generally follow the ISO/TR 20432 guideline on durability, but could have different names, depending on the design standard or guideline used as shown in Table 2.

Table 1 - Function-related characteristics and test methods to be used for the reinforcement function

Characteristic	Test method
Tensile strength	EN ISO 10319
Elongation at maximum load	EN ISO 10319
Stiffness at 2 %, 5 % and 10 %	EN ISO 10319
Tensile strength of seams and joints	EN ISO 10321
Friction	EN ISO 12957-1 EN ISO 12957-2
Tensile creep	EN ISO 13431
Damage during installation resistance	EN ISO 10722
Durability	According to Annex B

Table 2 - Characteristics and corresponding reduction factor

	Characteristic	Standard or guideline: ISO /TR 20432	Standard or guideline: EBGEO Germany
Mechanical behavior	Tensile creep	RF_{cr}	A_1
	Damage during installation resistance	RF_{id}	A_2
	Tensile strength of seams and joints		A_3
	Dynamic effect		A_5
Chemical durability	Resistance to hydrolysis	RF_{ch}	A_4
	Resistance to oxidation	RF_{ch}	A_4
	Resistance to weathering UV	RF_w	

2 Product characteristics for basal reinforcement

2.1 Geosynthetics in basal reinforcement

The main tasks of geosynthetics in basal reinforcement are to carry the load from the structure that the subgrade cannot; to enhance arching, to control differential settlements and resist lateral thrust of the embankment. Ultimate strength is crucial for Ultimate Limit State (ULS) analysis as defined in Eurocode 7 EN1997. Additionally, strain criteria's are imposed either in direct Service Limit State (SLS) analysis or by limiting deformation in ULS analysis. That means that the stiffness is an important parameter, in addition to the strength.

2.2 Embankments on soft soil

The design of embankments on soft soils use slip circle failure models in the ULS. The main property to determine is the ultimate tensile strength. However, the serviceability may require a limitation of the geosynthetic deformation. Therefore, the choice of the reinforcement will be based also on the stiffness behaviour of the product.

2.3 Embankments over piles

In piled embankments, the load transfer from the embankment to the piles and the differential settlements between the piles depend on the geosynthetic deformation. The

Abstract
Basal reinforcement should be designed following accepted design guidelines or standards. These guidelines all use calculation models that end up requiring a minimum strength and stiffness of the material in time. The production technology in itself is not an issue in the choice of the product applied. This publication highlights the relevant characteristics of basal reinforcement, their influence on the design and, if the production technology does matter, how to achieve the needed performance.

main properties to design are the geosynthetic stiffness and the tensile strength. Scaled model experiments on piled embankments, carried out by Van Eekelen et al. (2012), did not highlight significant differences of performance between geotextiles or geogrids having the same mechanical characteristics. This is shown in Figure 1: load part A is the load transferred to the piles directly and load part B is transferred via the geotextile or geogrid towards the piles.

The new CUR226 guideline on basal reinforced piled embankments (CUR226, 2016, see Van Eekelen (2016)) adopted a calculation model that has been validated with measurements on piled embankments where reinforcement layers of geogrids were applied, sometimes combined with a woven (geogrid on geotextile). The design guideline however accepts piled embankments with high strength wovens on top of each other, if additional measurements and/or suitability tests have been conducted in representative practical cases with which it have been demonstrated that the system comes within the framework of the design guideline. Such measurements will be carried out this year on a job site where the behavior of both the structure and the geosynthetics will be measured and analyzed by an independent institute.

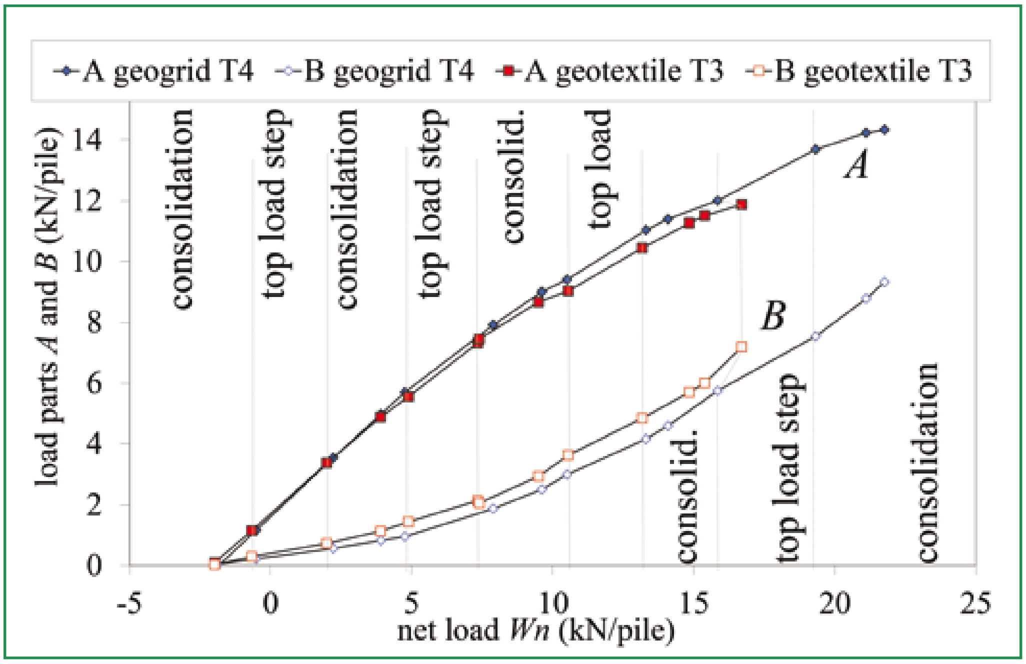
2.4 Embankments over potential cavities

In this application where the geosynthetic will bridge possible voids under the embankment, limited deformation at the surface is the major requirement. The corresponding geosynthetic strain varies depending on the geosynthetic stiffness and the thickness of embankment relatively to the size of the void. When a cavity grows up to the top of the subsoil, the first task of a geosynthetic consists of maintaining the structure above, by combining separation and reinforcement. Separation is needed, because any part of the fill falling through the geosynthetic, will result in more deformation of the structure above. Only geosynthetics with a small opening size, such as wovens or composites, are able to separate and are suitable when used at the base of the reinforced structure.

3 Characteristics and geosynthetics type

3.1 Ultimate Tensile Strength

The Ultimate Tensile Strength (UTS) is the strength at failure of the geosynthetic and depends on the raw material and the quantity used. Using weaving or knitting techniques, very high strength geosynthetic, i.e. above



Source: Van Eekelen et al., 2012.

Figure 1 *- load transmitted to the piles (part A) and to the geosynthetics (Part B).*

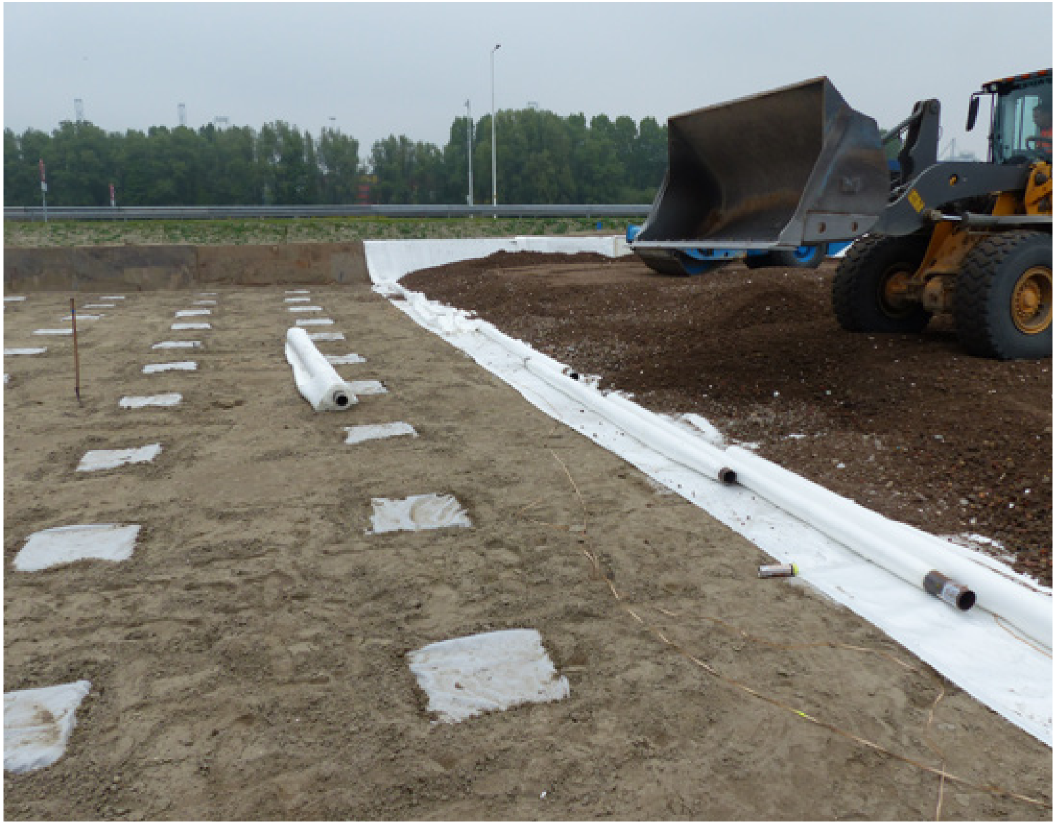


Figure 2 *- Piled embankment for the A15 constructed with high strength woven geosynthetics.*

Figure 3 - *High strength woven geosynthetics preventing possible damages to a high speed train line in a French cavity-sensitive area.*

Table 3 - Interaction at different interface – Kiwa test report (2015)

	Interface Friction angle δ	Interaction coefficient α
PET Woven 400/50 vs sand (ϕ=38°)	31.5°	0.78
PET Geogrid 400/30 vs sand (ϕ=38°)	31.9°	0.79

2000 kN/m can be manufactured.

3.2 Strain strength curve, isochronous curves and stiffness

The stiffness characterizes the capacity of a geosynthetic to resist to the deformation under load. It depends on the raw material, the quantity of material and the geosynthetic construction. It does not matter if one raw material is stiffer than others, as long as the final product fulfills the stiffness requirement in the design.

3.3 Interaction

Interaction involves two mechanisms: sliding of the soil mass on the geosynthetics or vice ver-sa and pullout of the geosynthetic in the anchorage zone. Interaction between geosynthetic and soil depends on the type of geosynthetic,the soil grain size distribution and the soil strength. The interaction of geogrids with adjacent soil is determined by a combination of end-bearing and surface friction whereas that of woven geotextiles is by surface friction alone.

However, endbearing occurs only if the aperture size is sufficient. For high strength ge-ogrids, (eg tensile strength above 400 kN/m), interaction may occur mainly by friction and may not differ strongly from geotextiles, wovens or composites. Table 3 shows that the difference may be negligible with fine granular soil such as sand or material containing fines.

4 Conclusion

The use of geosynthetics in the basal reinforcement of embankments is a common technique today. The design of the reinforcement relies on accurate design methods meant for the applica-tion such as embankments on soft soils, embankments on piles or bridging voids.
European standards define the design rules, provide the required level of safety and specify the decisive characteristics of the geosynthetic . Geosynthetics have to be chosen for their capability to fulfill these specifications.

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ADVERTORIAL

For more than 65 years Genap has specialized in geo-synthetic applications in the horticultural and agricultural sectors and for civil projects. At our factory in 's-Heerenberg in the Netherlands, where the laboratory and design office are also located, the customized geo-synthetic solutions are produced. Our construction teams are deployed worldwide for on-site installation. The highest level of quality can be guaranteed as we undertake everything ourselves from solution design to installation. Since 2015 Maji Water Storage Ltd, our sister company in Kenya, produces, supplies and installs water storage systems for the Eastern African market in Nairobi.

Photo 1 **-** *The installation of the geo-membrane at Lake Boyuk Shore*

Dam covers Lake Boyuk Shor, Azerbaijan

Engineering, local prefabrication and under water installation of waterproof geo-membranes

To ensure an untroubled lake view during the European Games held in the new Olympic Stadium in Baku Azerbaijan in 2015, the polluted water and soil of Lake Boyuk Shor caused by waste of the petrochemical industry, needed to be cleaned up. The lake was split up in several parts by building new dams while the sludge was dredged and transported to a nearby location for treatment and purification. To avoid the risk of seapage of contaminated water through the dams into the sanitized area, Genap installed a locally prefabricated geo-synthetic barrier with an oil resistant XR-5 geo-membrane.

Landfills in Wuhan, China and Erbil, Iraq

Supply and Installation geo-synthetics for landfill

For the first landfills in the Chinese city of Wuhan and in Erbil, Iraq, Genap supplied the necessary geo-synthetic products and supervised the installation. Both landfills were constructed as municipal solid waste sites with a compartment to store hazardous waste and liquids. A lagoon was constructed to store percolation water to be collected from the landfill by means of a HDPE piping system after which it is treated for purification reasons.

Photo 2 **-** *HDPE-bottom lining, Erbil Landfill, Iraq*

Enclosed reservoir for irrigation water, Al Ain, United Arab Emirates (UAE)

Installation of a fully enclosed water reservoir

For a tree nursery in the UAE Genap installed a fully enclosed water reservoir for the purpose of irrigation water. At our facility in The Netherlands the panels of the PVC geo-membrane were prefabricated and welded to one sheet on site.

Considering the high level of UV radiation a special fiber-reinforced PVC foil with a blackout layer was used. This completely prevents formation of algae growth caused by UV radiation. In addition, the top cover prevents the water from becoming contaminated due to external influences (sand storms for example) and evaporation caused by extremely high desert temperatures.

Locations

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Photo 3 **-** *The fully enclosed reservoir for irrigation water in Al Ain, UAE*

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Witteveen+Bos Consulting Engineers, The Netherlands

C. (Charlie) Angenent
Genap geomembrane systems, The Netherlands

J. (John) Scheirs
ExcelPlas Polymer Testing Services, Australia

Geomembrane systems in The Netherlands and abroad
risks and lessons-learned

Figure 1 - *Slope failure by stability problems after finishing the geomembranes caused by external water pressures versus insufficient backfill levels [Genap]*

Table 1 - Risk assessment based on project phases

Phase	Risk issue
Design	• Geomembrane material selection. • Maximum design water levels during construction and completion stage. • Feasibility of local excavations within the geomembrane. • Presence of environmental pollutions. • Stability of slopes/retaining structures, especially in case of stacking several geotextiles. • Feasibility re-use local excavated material around the geomembrane. • Connection type/detailing to structures.
Construction	• Quality of welding (on site/off site). • Weather conditions (rain, sunlight, UV-radiation, frost, wind conditions, etc.). • Installation damage by handling personnel, equipment or vandalism. • Stability problems caused by external water pressures, loads, etc. • Suitability subsoil (i.e. subgrade) to apply geomembrane. • Backfill method and backfill material quality. • Local excavation within backfill material (drainage pipes, sewage, etc.).
Completion and Maintenance	• Damage by a calamity with fire or aggressive liquids. • Gradual damage by external pollutions from the containment or adjacent sites. • Damage/puncturing geomembrane by human activities (drilling, digging, foundation works, etc.). • Lack of maintenance to drainage and sewage systems.

1 Introduction

The success of realizing a watertight and durable geomembrane construction depends on a lot of factors. In terms of risk analysis we can divide the construction process in three important stages:

1. Design stage
2. Construction stage
3. Maintenance stage

A technical risks assessment based on these project phases is given in table 1. The common denominator in all three stages will be the limited knowledge of geomembrane systems, material properties and procedures. A lack of knowledge means that potential risks are not being recognized at an early stage, resulting in a possible major impact. An example of severe damage by a slope failure is given in figure 1.

2 Risk assessment

Direct or gradual developing damage of the geomembrane can cause leakage and if not controlled total failure of the construction may occur. It shall be evident, severe damage or total failure should be avoided by recognizing the risks at an early stage.

For risk management technical risks are to be classified. Each identified risk can be rated to the occurrence probability and impact effecting costs and time. Also precautionary actions and residual risks are assessed. For obtaining an active risk management during the building

Abstract

This article gives an overview of risks and lessons learned about geomembrane constructions in The Netherlands. The aim of this paper is to emphasize an integral approach during the entire process and importance of acknowledging quality risks to all involved companies. The success of a watertight and durable installation will depend on an integration of design aspects, materials, construction issues and quality assurance. To avoid risks during lifetime attention shall be given to proper restrictions, maintenance issues and monitoring of leakage/durability. The article presents examples, to illustrate risks and lessons learned.

This article is supporting a specialty session at the Eurogeo6 congress Ljubljana (Slovenia) in September 2016. A more extensive version can be found as official paper to the congress proceedings.

process, it is important the risk assessment is lively and updated, as risks can change during the construction process. For example the occurrence of new risks or change in classification based on new circumstances.

3 Evaluation of risks

In the paragraphs below 5 issues are described more in detail and illustrated with pictures from engineering, construction and inspection practice.

3.1 Welding quality

Welds and seams can be designated as the weakest points of a geomembrane. According to *Scheirs 2009* seams are regions with high stress concentration due to defects in seaming operations, the heat-affected zone (HAZ) and residual stresses. The welding quality depends on a lot of factors, knowing the presence of contamination on the geomembrane welding surface (soil/moisture), weather conditions, workmanship of the personnel involved, material selection, and proper inspection. In figure 2 to 3 several examples are given of defects designated at welds and seams.

Obtaining a high quality weld will require serious craftsmanship and experience. Factors to take into account to ensure welding quality are weather conditions (humidity, low and high temperature, abrupt changes of temperature, etc.). Wind, rain and pollution by means of mud or sand (blown by the wind into the joint) will also influence the quality of the welding. Also good maintenance of the welding machines is important to insure proper welding. The machine shall be cleaned after usage and it shall be free from any pollution to the heating and pressure rollers.

3.2 Suitability excavated and re-use soil material

A lot of geomembranes are used and installed as underground barrier in the subsoil. This will imply excavations and backfill with soil/stone material, resulting in damage risks to the geomembrane. Based on electrical leak location surveys performed by *Nosko et al. 1996* it was assessed that about 20% of the leaks occur at seams (improper welding), but over 70% (!) of leaks occur when the liner is covered by soil or stone. Based on these values and practical experiences the quality of groundwork's is showing a factor which is under-estimated in a lot of projects. The covering operation has to be seen as a very critical stage for the geomembrane. Related to this issue many examples in projects are known, observing unacceptable circumstances during the excavation or covering stage (see figure 4).

Most desirable material to use around a geomembrane will be clean sand suitable as infrastructure foundation, according values of the standard contract specifications used in the Netherlands (CROW, 2015). Furthermore, the soil material adjacent to the geomembrane

Figure 2 **-** *Example of poor nip roller tracking in a HDPE wedge weld [Scheirs]*

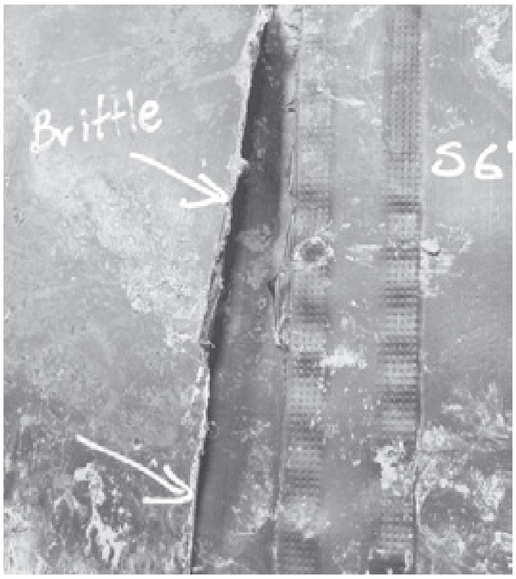

Figure 3 **-** *Example of stress cracking on edge of overheated weld [Scheirs]*

Figure 4 **-** *The presence of sharp/big stones without protective geotextiles will significantly increase the risk on damaging the geomembrane [Gerritsen]*

***Figure 5** - Example of proper installation using a protective non-woven geotextile at the subsoil and topside of the geomembrane, working sequence with excavation works and installing method at large slopes [Gerritsen]*

shall be free of admixtures with sharp stones, desiccated subgrade, boulders, concrete blocks, tree roots, construction waste (wood, beams, steel rebar, nails, piles, drainage or sewage pipes, foundation materials etc.).

The subsoil on which the geomembrane will be installed needs also to be clear from sharp subjects which may puncture through the geomembrane. Measures can be taken for additional protection of the geomembrane. Since 2012 state-of-art geomembrane applications in The Netherlands are provided with protective non-woven geotextiles on both sides (bottom/above), see figure 5. In case of non-woven protective geotextiles attention should be given to the material type (polypropylene / polyester), minimum density, dynamic cone drop resistance and static puncture resistance. Applying a good quality non-woven will reduce the risk of perforations of the geomembrane significantly.

3.3 Stability of slopes, backfill and retaining structures

To ensure a safe situation during the design and building process a lot of attention should be paid to the stability of slopes, backfill and retaining structures. In case stability is not assured, this will have major effects to the project. In case of (geotechnical) slope failure adjacent to geomembrane constructions damage to sealing is almost inevitable. Worst case this can result to a total loss of the geomembrane sealing and major effects to costs, planning, etc.

An important attention point is the presence of groundwater affecting the stability of slopes and retaining structures. Groundwater levels and overpressures used for submerging geomembranes can have a significant effect on the stability of geomembranes and the deformation of retaining structures, used in cases of limiting space by vertical boundaries and fixations of geomembranes *[Gerritsen, et al., 2014]*. The conditions of groundwater are of major importance, and also the proper working of dewatering devices (deep wells, filters, etc.). In case the dewatering devices (pump generators) fail, a quick build up of water pressure can occur, this can cause severe problems to the stability of geomembrane construction. Also excessive rainfall can lead to problems by rapid increasing groundwater levels, exceeding the design water levels or overflow of the working area. Several cases are known of groundwater conditions which damaged the excavated slopes or already installed geomembrane structure during the construction process (see figure 6).

3.4 Installation damage

During the construction stage the geomembrane remains very sensitive for damage, knowing it can be exposed to all influences from transport, handling, weather influences, etc. During transportation and off loading full rolls may be damaged by means of insufficient packaging or due careless handling by drivers of fork lifters or cranes. Another cause is the impact of heavy equipment used on the construction site, causing mechanical damage by cranes, trucks, dozers (see figure 7).

3.5 External influences after completion

After completion of the installation it may seem

Figure 6 - *Geomembrane 'whales' due to uplifting water pressure below the geomembrane related to improper backfilling levels [Genap]*

Figure 7 - *Mechanical damage to a geomembrane anchoring trench by using wrong backfilling equipment (crane) and less instructed personnel on site [Gerritsen]*

that the geomembrane is save for lifetime. However, during the entire lifecycle all kind of influences can harm a geomembrane. Special influences to be listed are:

1. Damage of geomembrane during lifetime by a calamity with fire or aggressive liquids.
2. Gradual damage by external pollutions from adjacent sites.
3. Damage/puncturing geomembrane by human activities (drilling, planting trees, etc.).
4. Lack of maintenance to drainage and sewage systems.

Damage/puncturing geomembrane by human activities is a likely cause for damage. The existence of a barrier is often 'forgotten'. Examples can be drilling, digging and foundation works.

4 Conclusions

The success of realizing a watertight and durable sealing will depend on a good understanding of design aspects, materials and on quality assurance during the building process. Risk determination shall be in cooperated in the total process. In terms of risk analysis we can divide the construction process in three important stages: the design, construction and maintenance stage. The common denominator in all three stages mentioned above will be the limited knowledge of geomembrane systems, material properties and procedures. A lack of knowledge means that potential risks are not being recognized in an early stage resulting in a possible major impact to final harming the integrity of the geomembrane construction during the building process or even years after completion.

An integral perspective is necessary to obtain a durable geomembrane construction. For the construction stage it can be stated that open and active communication between all involved parties (client, contract manager, supervision staff, engineer, main and subcontractors, QA) is of major importance. Special attention shall be given to the interfaces of different disciplines by main and subcontractors.

The installation of geomembranes as underground barrier in the subsoil will imply excavations and backfill with soil/stone material, causing a strong interface with damage risks to the geomembrane. The covering operation has to be seen as a very critical stage for the geomembrane. Related to the high risks of covering, the best standard should be geomembranes to be embedded by non-woven protective geotextiles at all times. Applying a good quality non-woven will reduce the risk of perforations of the geomembrane significantly by external influences as well during the construction stage as well after completion.

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ADVERTORIAL

New development; the valuable use of GPS based logging systems for vertical drain installation

PVD, also known as wick drains, are one of the most commonly used techniques to make soft compressible subsoil with a low bearing capacity constructible by accelerating the dissipation of excess pore pressure. In the recent years a sound, GPS based, logging system has been developed by contractor COFRA, Amsterdam. The system registers amongst others, the drain coordinates, push forces at selected intervals and maximum depth of the vertical drains. The system has proven itself to be of added value for both the contractor as well as the client/engineer and can be used on projects upon the clients request.

As the GPS system is capable to load AutoCAD drawings and make these visible to the excavator operator, valuable information can be provided to them. These include the installation grid, locations of underground infrastructure, obstructions or levels of installation, limitations on height and other restrictions. It can also be used to install highly optimized vertical drain areas with complex geometries, as shown in Figure 2. In this way the GPS system contributes to limitation of operational risks.

The processed data can be used to:

- Generate a depth map of the installation data. This can be used to map the bedrock or refusal layer at a 1to 2m horizontal interval. See Figure 3 for a map of a bedrock layer of a project in Oslo. In this project the initial dynamic probes, locations are marked as white diamonds in the figure, showed a variable image of the installation depth and top of bedrock layer. The maximum depth obtained from the probes coincided well with the installed depths of the PVD. However, during installation it was found that the top of the bedrock level varied even more than anticipated and a large section had a very thin clay layer. This high bedrock level was missed by the site investigation campaign. This saved the contractor the placement of surcharge and optimized the settlement monitoring design

- Map intersecting layers with a higher or lower required push force by generating a cross section of push force:. Figure 4 shows a cross section made using the push forces of a project in Rotterdam. The purple colors indicate dense sand layers with cone resistances of over 10 MPa. The two blue colored section between -5m and -15m indicate soft compressible clay layers. It is also visible that there are more clayey sections present in the tidal deposit starting from -15m below the surface. This example shows that being able to plot the push force in a cross section can be valuable as it can be used to map geological features and limit geotechnical risks.

- Check if the design is followed and where deviations from the design occur. In the example shown in Figure 3 it is shown that a deviation is found, so that measures in the monitoring and final surcharge heights can be taken

- Evaluate the positions of the settlement markers. On the section with the high bedrock

Figure 1 **-** *Example of GPS based installation of PVD without the use of pre marking on the ground.*

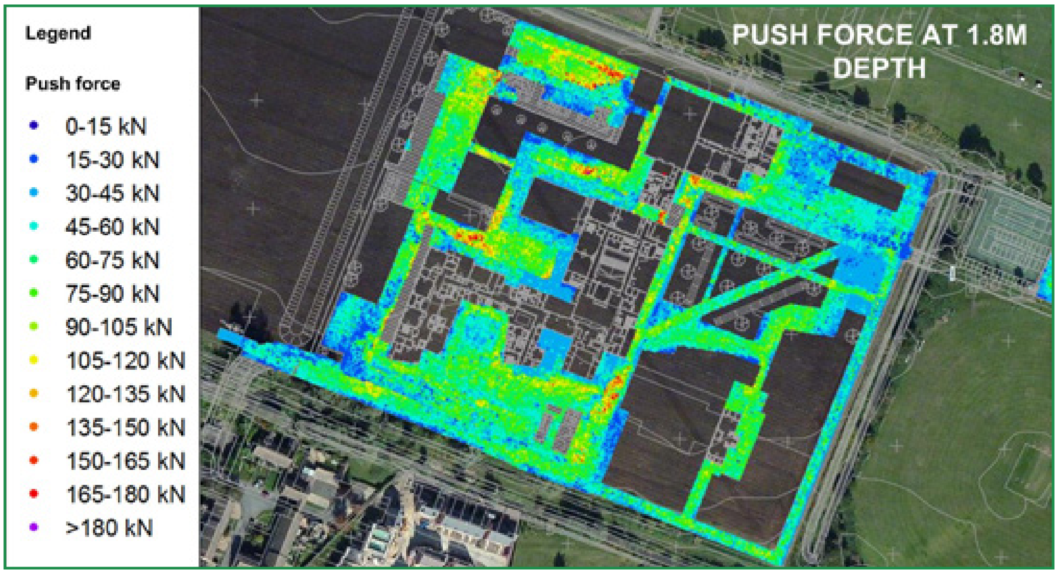

Figure 2 **-** *Map showing the push force at a specific depth in a highly optimized and complex installation grid, only possible to execute with GPS based logging and positioning.*

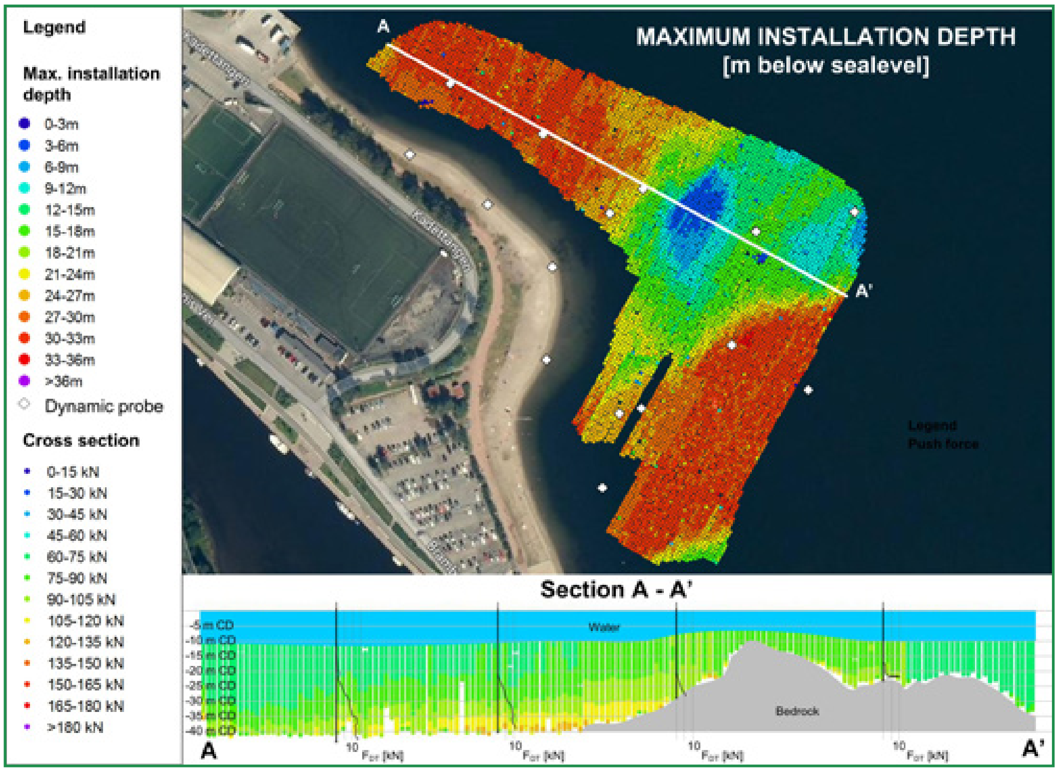

Figure 3 **-** *GIS compilation of the logger data showing maximum installation depth and a cross-section through section A-A'.*

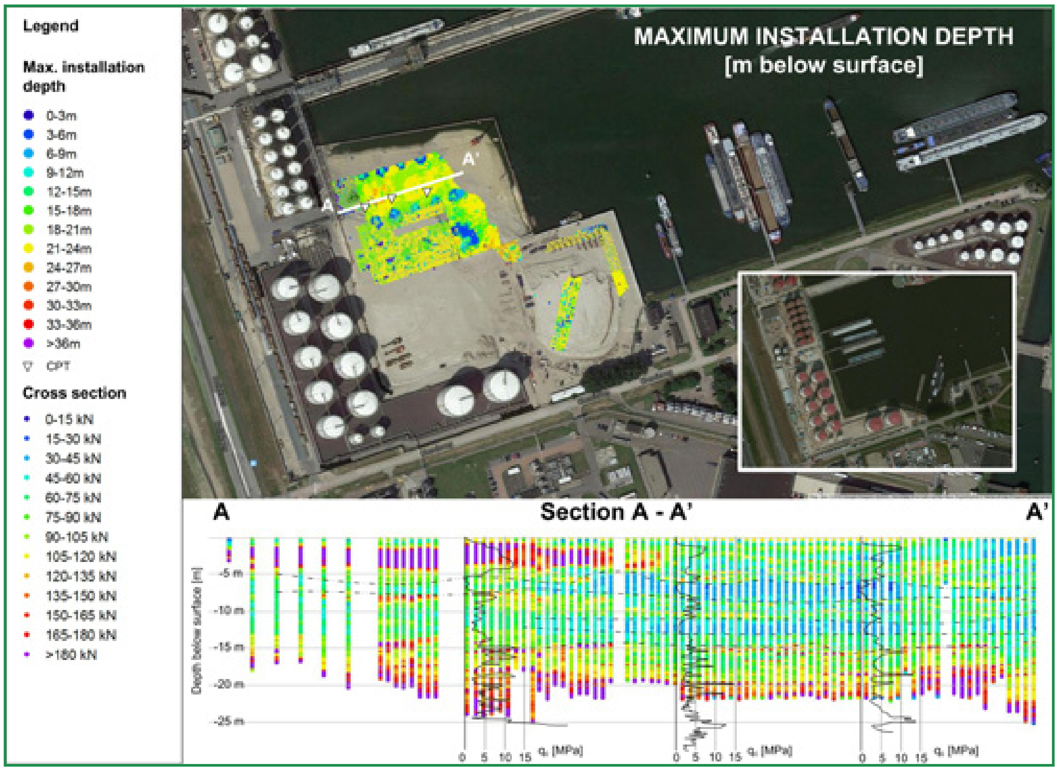

Figure 4 **-** *GIS compilation of the logger data showing maximum installation depth and a cross-section through section A-A' .*

level, additional markers could be placed and surcharge could be reduced.

- Map obstructions. In figure 4 small blue circles are visible at the location of the piles of an old jetty. The piles were removed after the placement of the fill by pulling and vibrating. Due to the vibrations the sand at the old pile locations was densified significantly to such levels that the light rigs were not able to push through. These locations were afterwards predrilled to make the installation of the PVD possible and could in the fufutre also be provided to the operator to prevent unexpected refusal or indicate specific risk zones.

- Plot the push force at specific depths. Figure 2 shows the push force at specific depth. This can be used to map sand lenses or other geological or man-made features that are present in the subsoil.

Conclusion / Resume

As the GPS system is capable to load AutoCAD drawings and make these visible to the excavator operator, valuable information can be provided to them. With this pre-marking of installation points at site is no longer needed. Furthermore site restrictions are always available for the excavator operator at his computer screen, thus contributing to limitation of operational risks on wick drain installation.

The GIS images presented show that the use of GPS based logging can provide additional information to the geotechnical engineer and the client, especially when stiff clay layers or sand layers are present. With the use of this system, push force profiles can be constructed which can be used to map geological features. It can also be used to generate an overview of the installation depths over the installation area. Both can help during the monitoring phase in the assessment of the lateral variations within the site, placement of the settlement markers, piezometers and ultimately limiting differential settlements by adjusting fill heights. This makes monitoring more specific and reliable, thus optimizing operations and limiting operational risks in the consolidation phase

In the building process the subsoil is always a factor of risk, no matter the number of soil investigation that has been done. Working with the GPS system in the critical wick drain installation phase contributes to limitation of risks as well as optimization of design. Introducing GPS is for Cofra one of many steps they have made over years in transforming wick drain installation worldwide to a professional level.

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Tensar | THE COMPANY YOU CAN BUILD ON™

TensarTech® Stratum™ provides the 3D solution for building over soft soils

The soils in a lot of projects are challenging. Often the subsoil has a poor bearing capacity, so is not able to withstand the (crane) loads and will squeeze the weak subsoil sideways while building railways, motorways, dams and heavy duty platforms. The solution is a TensarTech® Stratum™, as this is a rigid foundation composed of a 3D cell construction of geogrids, filled with granular material.

The effects are maximum utilization of the ultimate-bearing capacity of the soil, making it possible to construct in one go, to act as a drainage layer for e.g. consolidation water and reduce differential settlements

Some of the common solutions to build over soft soils are piled-raft foundation systems, removal and replacement of the soft soils, adopting staged construction and using ground improvement technology like stone columns or soil-cement columns, etc. Analysing these solutions the geocell solution will be found to be both technically very suitable and also considerably cheaper than the other solutions.
Already in 1983 became the first "Geocell" designed and built. Many projects in continental Europe are built since them with our TensarTech® Stratum™ as a very effective high 3D cell structure for foundations under embankments and crane platforms with controlled settlements.

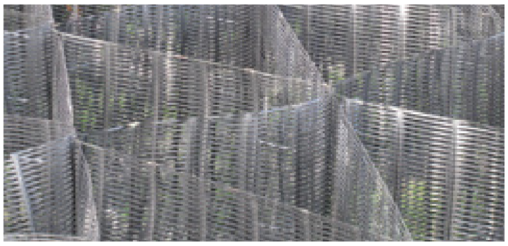

Figure 1 **-** *3D Geocell structure*

The design of a "Geocell" is defined in the international BS8006-1, in which the operation is described clearly, and we have shown in the projects that we meet design requirements successfully. The geocell-structure is so effective because it spreads loads at a minimum angle of 1v: 2h, intersects possible slip circles and force them, due its stiffness, in deeper layers. Furthermore the rough underside ensures the utilization of maximum shear capacity of the weak layer.
Most of the projects are checked with an FEM-analyses (Finite Element Method, e.g. PLAXIS), plate pressure test and/or in-situ load tests. Monitoring of the settlements showed that real settlements are less as compared to the calculated settlements and more importantly the geocell provides even and controlled settlements. This controlled settlement shows that the geocell mattress forms a stiff and stable working platform by effectively maximizing the pressure distribution of applied loading onto the soft foundation soils. Therefore the TensarTech® Stratum™ is often build in working platforms as a base for the crane area in wind farms. Even when the geocell foundation is there to provide a working platform for a piling rig, it´s very easy to incorporate provisions for applying, cq. driving true piles or other objects.

In practice

The filled geocell offers a direct working platform for heavy machinery and traffic over the weak subsoil. Therefore it is possible to carry out projects faster and be sure that construction stays within the desired timeframe. For example the Dutch project on the A7, Sneek. Several geocells with a total plan view area of approx. 6,000 m² were built and filled in just 13 working days. Furthermore the use of geocells made it possible to meet the stability requirements and to enable the build of a soil body construction to be carried out within the desired timeframe. On the low strength 2 meter layer of peat (C_u = 15 kPa, c = 3,3 kPa, ϕ' = 14,7°) it was possible to build with a lift of 0.60 m per week
Another (build) example is the construction of the windmill park NOP where the windmills have heights over 135 meter and therefore the crane they have to use is the biggest in Europe. Because of a weak clay layer in the subsoil the use of the crane was not possible on the existing underground. In this project Tensar provided the geocell solution and saved (according to the contractor) up to 25,000 truck transports and 4,000 tons CO_2 compared to a pilled foundation.

TensarTech®Stratum™ will give geotechnical benefits, namely;

- rapid performance by maximizing the capacity of the weak subsoil,
- increasing the capacity by large spread of charges,
- forcing critical circles sliding into deeper layers and
- through the stiffness of the geocell controlled and uniform subsidence.

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